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LATERAL FORCES OF EARTHQUAKE AND WIND

By a Joint Committee of the San Francisco, California Section, ASCE, and the Structural Engineers Association of Northern California

STRUCTURAL DIVISION

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PAPERS

LATERAL FORCES OF EARTHQUAKE AND WIND

By Arthur W. Anderson;[1] John A. Blume,[2] M. ASCE; Henry J. Degenkolb,[3] Assoc. M. ASCE; Harold B. Hammill,[4] M. ASCE; Edward M. Knapik,[5] M. ASCE; Henry L. Marchand;[6] Henry C. Powers;[7] John E. Rinne,[8] M. ASCE; George A. Sedgwick,[9] Assoc. M. ASCE; and Harold O. Sjoberg,[10] Assoc. M. ASCE

Synopsis

In this paper the writers have drafted a lateral force provision which constitutes a reasonable requirement for building codes. Although it was prepared specifically for California conditions, the code is generally applicable, possibly with some quantitative modifications to suit different local conditions. The report includes the development of a more rational criterion for design seismic forces than has been attempted heretofore in lateral force codes.

Introduction

Earthquake design forces stipulated in California codes vary from a maximum of 13.3% of vertical loads for the Los Angeles, Calif. formula to a minimum of 2% of vertical loads for the state law known as the Riley Act. Criteria by which these forces are established are generally empirical and not always consistent or rational. They do not account for the dynamic characteristics that determine how structures respond to earthquake ground motion. Advances in scientific knowledge have not been reflected in current building codes.

Note.—The authors of this paper constitute a Joint Committee of the San Francisco, Calif. Section, ASCE, and the Structural Engineers Association of Northern California, as appointed April 2, 1948. Mr. Rinne served as Joint Committee chairman. Written comments are invited for publication; the last discussion should be submitted by October, 1951.

[1] Corlett and Anderson, Archts. and Engrs., Oakland, Calif.

[2] Cons. Structural Engr., San Francisco, Calif.

[3] Structural Engr., J. J. Gould, Cons. Engr., San Francisco, Calif.

[4] Cons. Civ. Engr., San Francisco, Calif.

[5] Partner, Huber & Knapik, Cons. Engrs., San Francisco, Calif.

[6] Cons. Engr., San Francisco, Calif.

[7] Structural Engr., H. J. Brunnier, Cons. Engr., San Francisco, Calif.

[8] Civ. and Structural Engr., Standard Oil Co. of California, San Francisco, Calif.

[9] Structural Engr., W. P. Day, Archts. and Engrs., San Francisco, Calif.

[10] N. H. Sjoberg & Son, Gen. Constrs., Oakland, Calif.

On April 2, 1948, the San Francisco (Calif.) Section, ASCE, and the Structural Engineers Association of Northern California appointed a Joint Committee on Lateral Forces to evaluate the current understanding of wind and earthquake forces and to draft a model lateral force provision for the building codes in California. The writers constituted the personnel of that Joint Committee. It is believed that, with some modification for local conditions, the resulting code (see Appendix I) will be applicable to a larger field.

As part of its study the Joint Committee has:

1. Reviewed the historical record of earthquake damage;
2. Noted the limited accurate recording of ground accelerations in earthquakes;
3. Extended the mathematical response of simplified structures subjected to known earthquake acceleration patterns into a practicable dynamic approach to earthquake design forces on engineering structures;
4. Indicated a number of problems that require further study, acknowledging that there is still much to be learned about earthquakes and earthquake effects on structures; and
5. Prepared the model lateral force provision.

A statement of the function of the lateral force provision is necessary so that it is understood and agreed what is to be accomplished by the provision. It is the province of the building code to specify design and construction requirements which will result in structures safe against major structural damage and the loss of life in the event of precedented winds or earthquakes. The lateral force provision is primarily structural. Panic and fire hazards, for example, are covered in other sections of the building code. The design requirements are considered to be the minimum consistent with the general objective. If unfavorable conditions of a particular site, such as the proximity of known faults, extreme exposure to high winds, or the importance of continued operation of a facility in the event of unprecedented winds or earthquakes, suggest the advisability of using design lateral forces in excess of those specified, this is left to the judgment of the owner on the advice of his engineer. The basis of such judgment is a matter of evaluating the calculated risk and is beyond the scope of the minimum provisions of a building code.

The relationship between wind and earthquake for design purposes is well recognized. Both wind and earthquake involve primarily lateral force considerations. Wind forces relate to areas, whereas earthquake forces relate to masses. However, the principle of determining the larger of the two forces and being governed thereby is accepted design practice, and is the basis for specifying both wind and earthquake minimum design forces in a code section entitled "Lateral Forces."

Wind forces have been studied extensively by others (see Appendix II); design criteria are well established and well in advance of corresponding criteria for earthquake forces. The principal problem of the Joint Committee was the determination of earthquake forces, and these, therefore, have been developed in greater detail than have wind forces. Considerations have been confined to the establishment of the design forces; no attempt has been made to specify methods of providing for these forces.

Earthquake History

References to the occurrence of earthquakes are sprinkled liberally throughout written history. To a lesser extent, observations of damage have been recorded, but few scientific records of ground motion have been made.

California has a comparatively short historical record. Some of the earliest records are the diaries of the padres and the early Spaniards which tell of heavy earthquakes in the 18th century. In Northern California alone, since 1800, there have been no less than seventy shocks of Intensity VIII or higher by the Rossi-Forel scale. Continued seismic activity, recently on a minor scale, portends future shocks which there is no reason to believe will be any more or any less severe than those experienced in the past. The earthquake of 1906 along the San Andres fault in the vicinity of San Francisco may be accepted as a reasonable maximum shock for design purposes in California. This earthquake is rated as one of the three strongest known to have occurred in California; but, unfortunately, the ground motion data are very limited and of questionable accuracy. The ground motion of more recent shocks, although of lower intensity, have been recorded more consistently.

In any event, there is no justification in the historical record for complacency or indifference to the design for earthquake forces. The brief seismic history of the United States as portrayed (54)[1], in Fig. 1, indicates that an awareness of the hazards of earthquakes and the application of the basic principles of seismic design should not be confined to California.

The year 1906 marks the beginning of a period of intensive study of earthquakes and research in earthquake-resistive design. Observations of the effects of the 1906 San Francisco earthquake have been well documented by several investigators. Records are also available of the following subsequent major shocks:

Location	Year
Tokyo, Japan	1923
Santa Barbara, California	1925
Long Beach, California	1933
Helena, Montana	1935
El Centro, California	1940
Fukui, Japan	1948

It is not necessary to relate extensively the recorded history of earthquake damage. A brief review of the nature of the damage will be given; more detailed information is available by reference to Appendix II.

Damage to Structures

Generally, the records confirm that buildings of sound design and good construction fare well and that those of poor design and poor construction, from the standpoint of lateral forces, fare badly. The advantages of flexibility coupled with strength were apparent to those reporting the 1906 shock and also subsequent shocks. Conversely, the hazards of rigidity coupled with struc-

[1] Numerals in parentheses, thus: (54), refer to corresponding items in the Bibliography (see Appendix II).

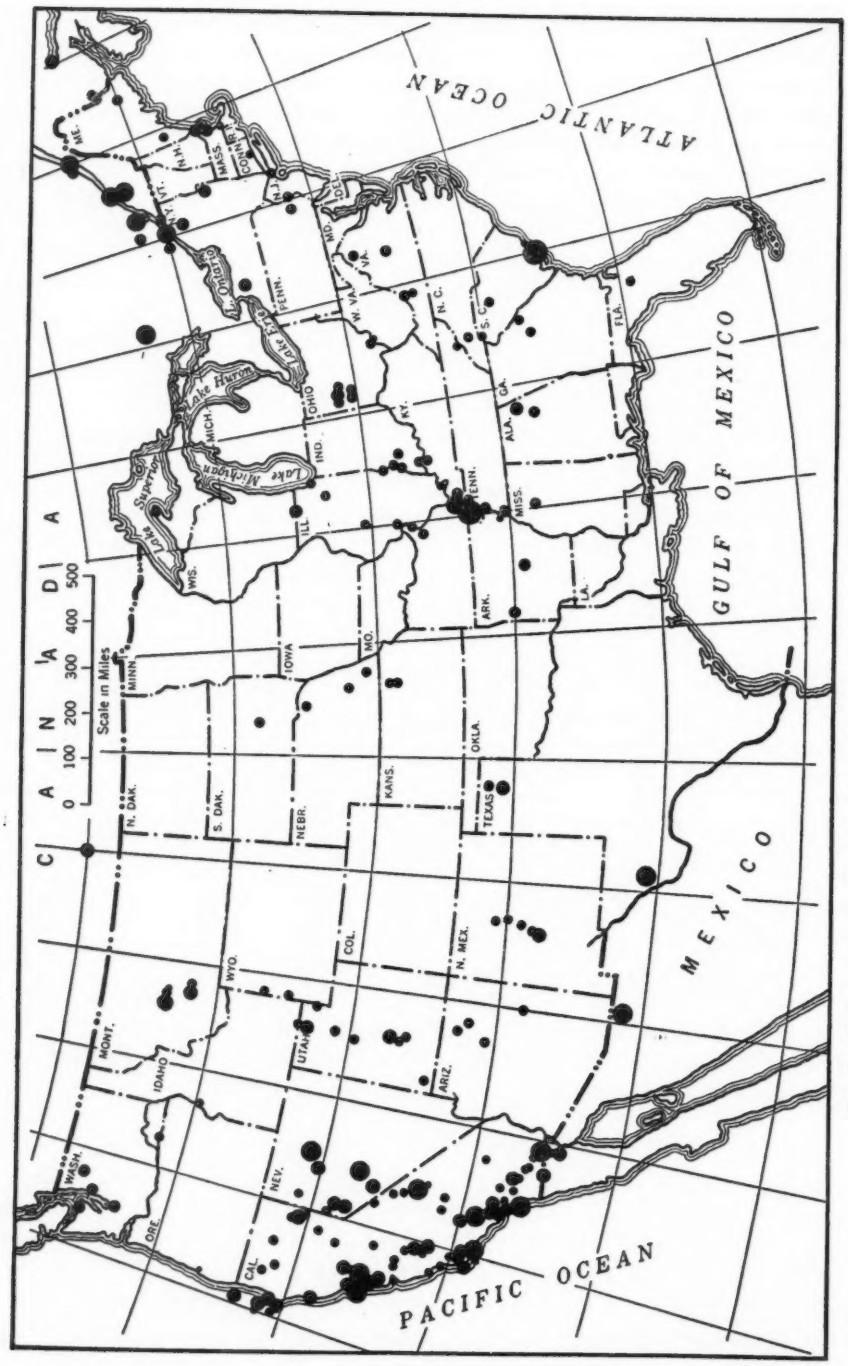


FIG. 1.—DESTRUCTIVE AND NEAR DESTRUCTIVE EARTHQUAKES IN THE UNITED STATES THROUGH 1946

turally weak materials and poor combinations of materials have been universally recognized and condemned.

Wood Frame Structures.—Little damage occurred to wood frame structures in 1906 except for broken glass, cracked plaster, fallen chimneys, and distortions caused by movement of filled or unstable ground. Fallen chimneys were the greatest hazard. This experience has been typical of other earthquakes.

Buildings of Masonry Walls and Timber Interior Framing.—Buildings of masonry walls and timber interior framing have had a very poor experience record. In 1906 this type of building suffered most heavily from the shock and was responsible for the greatest loss of life. Damage ranged from minor cracks to complete collapse and prompted the 1906 ASCE committee (55) to call this type of construction "hopelessly inadequate." There were examples, however, of satisfactory performance. The Palace Hotel, in San Francisco, suffered practically no damage, probably because it had numerous masonry cross walls and exceptionally good mortar. Agnews Hospital, near San Jose (Calif.), on the other hand, was a complete failure, with the loss of 112 lives. This disparity suggests the improper use of materials and poor design and construction. Subsequent earthquakes have demonstrated the same weaknesses in unit-masonry construction. Nevertheless, structures of this type can develop reasonable resistance to earthquakes, if special design and construction precautions are taken.

Buildings with Complete Steel Frames.—Buildings in which the frame supported all walls and floor loads gave a good account of themselves in 1906. Although varying degrees of damage occurred to walls, facings, and appendages, damage to steel frames appeared to be minor. The ASCE Committee on Fire and Earthquake Damage recorded one of the most authoritative observations of the earthquake effects (55).

Structures with X-bracing had some failures directly attributable to this rigid type of bracing. Those with moment-resisting frames were more successful, indicating the desirability of strength coupled with flexibility. The Ferry Building tower was X-braced with rods and suffered the greatest damage of any steel frame building.

The Claus Spreckels Building was braced with both portal and X-bracing, reportedly for a wind pressure of 50 lb per sq ft. Only minor damage was apparent in the steel X-bracing, although there was considerable damage to the stonework resting on the frame, especially betweeen the tenth and the sixteenth stories of this nineteen-story building.

Other steel frame buildings had less lateral resistance than the Claus Spreckels Building, generally in the form of portal or moment-resisting connections. Some buildings were large in area and had no special wind or earthquake bracing. Typical of these buildings were the Merchants Exchange, St. Francis Hotel, Fairmount Hotel, Monadnock, Rialto, and Flood buildings, in San Francisco. The damage to steel frames was negligible. Partitions made of tile cracked, but walls of brick, or stone and brick, did not suffer as much as might have been expected. No foundation damage was observed in any of these buildings.

Some rivets were sheared in the Flood Building, but this damage is generally regarded to have been minor. According to current methods of analysis, and with generous assumptions, this building will resist no more wind pressure than 10 lb per sq ft or a lateral load of 0.6% gravity.

The cracking of brick walls (St. Francis Hotel, Merchants Exchange, Atlas, New Chronicle, and Monadnock buildings, as examples) indicates a need, not only for an adequate bonding of the walls to the steel frame but also for a recognition of the rigidities of these structural elements.

The record in Tokyo in 1923 was somewhat similar, although evidence at hand seems to indicate that this earthquake was much more severe than the San Francisco earthquake of 1906. Quoting from a paper by John R. Freeman (*2a*),

> "There were 16 large steel-framed buildings in Tokyo. Of these, six were absolutely undamaged by the earthquake, the remaining ten being more or less severely damaged, one to the point of incipient collapse."

This last building, the Tokyo Kaikan, has been described as follows (*2b*):

> "The steel frame was very light. The steel columns were so small in cross-section and the bracing so little that this can hardly be classified as a steel frame building, the construction being far below both the Japanese and American standards for steel frame buildings."

The Nippon Kogyo Bank, in Tokyo, designed by Tachu Naito for a lateral force of g/15, was among the buildings practically undamaged.

Among the more severely damaged buildings of the steel frame type were the Marunouchi, Yuraku, Japanese Oil, Yusen, Mitsukoshi, Tokyo Kaijo, and Chiyoda buildings in Tokyo. Of these buildings, Mr. Freeman states, "* * * the question of bracing was more or less incidental, and probably subordinated to other considerations." From the meager details available, the undamaged steel buildings seemingly had reinforced concrete filler walls, whereas those that were damaged had brick or tile walls. Without more complete details, it is impossible to appraise properly the damage to these buildings.

Reinforced Concrete.—In 1906 reinforced concrete construction was in its infancy, but well-designed structures came through excellently. At Santa Barbara in 1925, reinforced concrete frame buildings with concrete floors and hollow tile filler walls suffered fractured concrete columns and more or less complete destruction of the tile walls. Other all-reinforced concrete buildings fared better, although there was some cracking of walls and piers. Tokyo contained a large number of concrete buildings in 1923. Of some 700 buildings of this type, approximately 75% were undamaged, 2% failed completely, and 23% were damaged in varying degree. The experience of composite construction with weak-bonded filler walls of another material, such as brick or terra cotta tile, has not been very satisfactory.

Building Appurtenances.—Taken as a whole, a very small percentage of buildings has suffered complete collapse. Considerable damage and loss of life, however, have been attributed to lack of attention to certain design and construction details. Fallen chimneys have been mentioned. Damage to parapets, cornices, filler walls, gable end walls, and various appendages has been

very common in all major earthquakes. Such damage is inexcusable, since, with reasonable attention to the details of the fastening of these parts to the structure, the damage and attendant injury and loss of life need not have occurred.

Structures Other Than Buildings.—Structures, such as bridges, towers, and chimneys, have also suffered damage. The records contain much data on da-

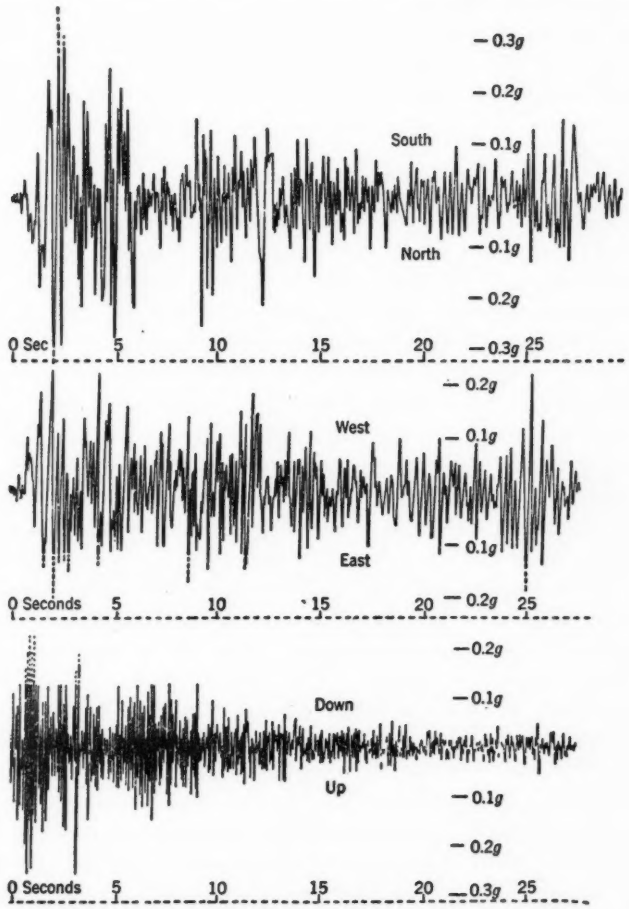


FIG. 2.—EL CENTRO ACCELEROGRAPH RECORD OF GROUND ACCELERATIONS; IMPERIAL VALLEY EARTHQUAKE OF MAY 18, 1940

mage to bridges which is not related here, since bridges are not generally built under the jurisdiction of the building codes. They are a subject for separate study and report. Industrial structures, such as freestanding towers and chimneys, however, do come under the building codes.

Tank towers of the rod-braced type have been extremely vulnerable to damage. One reason given is the relative rigidity of the X-bracing and its

relative weakness compared to the "reserve" strength of the vertical load-carrying members. Following failure of the rigid bracing, the tower deflects under the influence of the ensuing ground motion to the point where the remaining frame fails and collapses. Typical was the experience at El Centro, in 1940, where two elevated tanks collapsed, four tanks were seriously damaged, and two others were undamaged. The two undamaged tanks had been designed or rebraced for lateral forces of 0.1 g.

Frequent partial or complete collapse of chimneys (mostly brick, but also some concrete) has also occurred. An observation that the failures generally come well above the base indicates a weakness not properly covered in design by the conventional wind or static lateral force methods.

Ground Motion

Knowledge of the effects of earthquakes on structures is much more extensive than knowledge of the ground motion responsible for these effects. To account for the damage produced in San Francisco (1906) and in Tokyo (1923), for example, there are only rough estimates of the accelerations based upon observed effects on simple structures such as tombstones.

In 1932 the United States Coast and Geodetic Survey (USCGS) initiated a strong-motion program in California to obtain instrumental records of ground motion in destructive areas. The Long Beach Earthquake of March 10, 1933, produced the first results. They were imperfect, however, because the acceleration exceeded that for which the instruments were designed. The program was extended with instruments of more rugged design, and the El Centro record (51) of May 18, 1940, shown in Fig. 2, represents the most significant result to date. Table 1(*a*) summarizes instrumental data from Long Beach and Los Angeles, 1933; El Centro, 1940; and some of the aftershocks at Helena, 1935. Also tabulated in Table 1(*b*) are noninstrumental estimates of ground motion at San Francisco, Tokyo, and Fukui. The data in Table 1 should not be used without consulting the complete record cited in Col. 15. The velocities, accelerations, and displacements are maxima and do not necessarily correspond one with the other.

The strong-motion seismographs in general use are called accelerographs because, for all practical purposes, the record produced is that of three mutually perpendicular components of the ground acceleration. The acceleration record may be integrated to obtain the velocities and displacements against time which are shown in Figs. 3 and 4 for the El Centro record (51).

Aside from these instrumental records, the practice has been (and in the main continues to be) to rate earthquakes on the basis of their destructiveness, or on a so-called intensity scale. About 1883 the Rossi-Forel (37) scale was established, ranging through ten divisions of intensity. In 1931 the USCGS adopted the Modified Mercalli scale (48) ranging through twelve divisions, Intensity I being an earthquake barely felt under favorable conditions, and Intensity XII (the other extreme) being the most severe ever experienced. Even

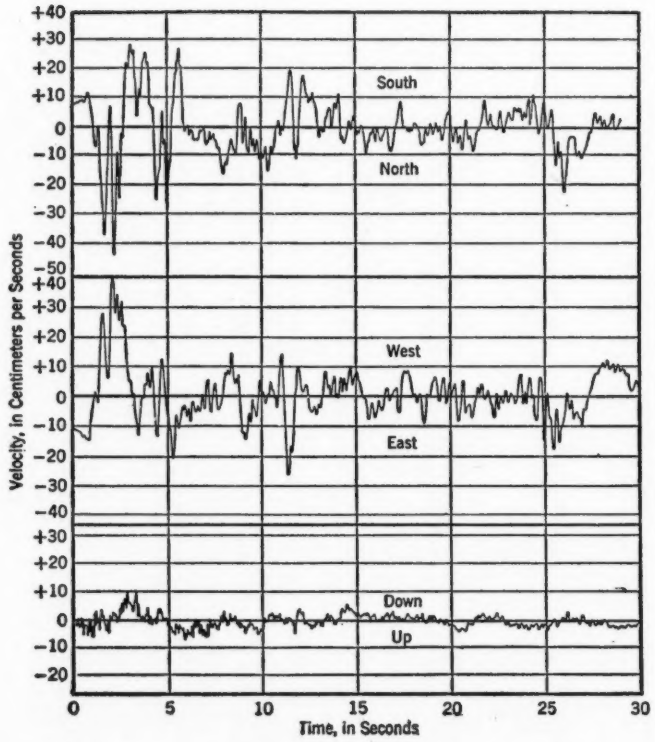


FIG. 3.—GROUND VELOCITIES FROM THE EL CENTRO ACCELERATION RECORD OF MAY 18, 1940

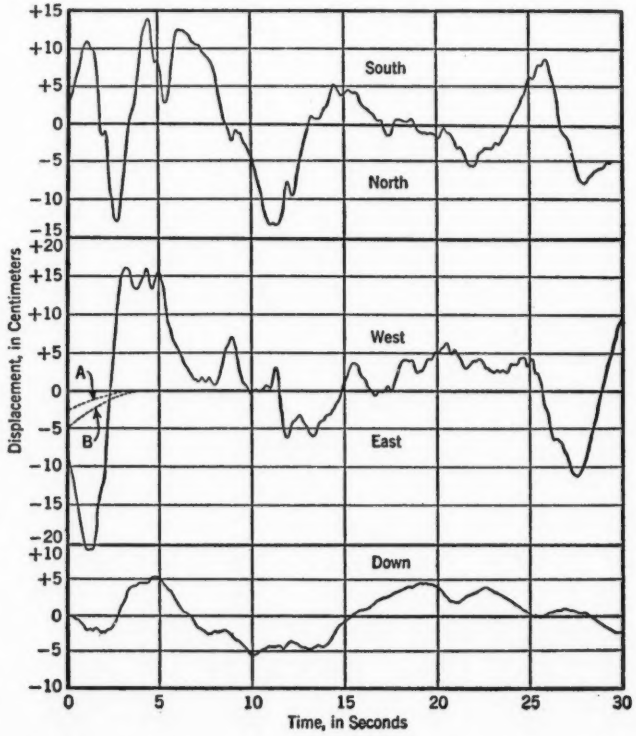


FIG. 4.—GROUND DISPLACEMENTS FROM THE EL CENTRO ACCELERATION RECORD OF MAY 18, 1940

where instrumental records of ground motion are obtained, the observation of damage and the establishment of the intensity of the earthquake by a damage scale is a necessary supplement to the records for an engineering understanding of the earthquake and its effects.

TABLE 1.—SUMMARY OF GROUND MOTION

Line	Description	Maximum Acceleration								Maximum Horizontal Velocity		Maximum Horizontal Displacement		References[a]
		Horizontal				Vertical								
		α $\left(\frac{\text{cm}}{\text{sec}^2}\right)$	$\frac{\alpha}{g}$	P Period (sec)	Displacement (cm)[b]	α $\left(\frac{\text{cm}}{\text{sec}^2}\right)$	$\frac{\alpha}{g}$	P Period (sec)	Displacement (cm)[b]	$\left(\frac{\text{cm}}{\text{sec}}\right)$	Period (sec)	(cm)	Period (sec)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
	(a) Strong-Motion Instrument Records													
	Long Beach, Calif., 1933:													
1	Utilities Bldg.[c]	230	0.24	0.30	0.50	250	0.26	0.11	0.07	25	1.50	6.0	1.5	(48a)
2	C M D Bldg.[c,d]	210	0.215	0.70	2.6	50	0.05	0.13	0.02	24	0.70	6.0	2.5	(48a)
3	Subway Terminal[e]	60	0.06	1.0	1.5	20	0.02	0.20	0.02	15	2.50	6.0	2.5	(48a)
4	Helena, Mont., 1935	115	0.12	0.13	0.05	78	0.08	0.125	0.03	9.6	1.2	1.8	1.2	(49a)
5	El Centro, Calif., 1940	320	0.33	0.3[f]	1.3	220	0.23	0.10	0.07	30	4.0	19.0	4.0	(50)
	(b) Noninstrumental Records													
	San Francisco, Calif., 1906:													
6	Rock	...	0.025	...	...	...	...	...	...	...	...	5	1.0	...
7	Marsh	...	0.30	...	...	...	...	...	...	...	...	10	1.0	...
8	Tokyo, Japan, 1923	...	0.33	...	...	...	...	...	...	...	...	18	2	...
9	Fukui, Japan, 1948	...	0.60	...	...	...	...	...	...	...	...	...	1.5	...

[a] Reference numbers corresponding to those in the Bibliography, Appendix II. [b] As calculated by the formula $\frac{\alpha P^2}{4\pi^2}$. [c] Long Beach. [d] Central Manufacturing District Bldg. [e] Los Angeles. [f] 0.3 sec to 0.5 sec.

EARTHQUAKE FORCES

Various assumptions have been used to determine, from the ground motion, the forces acting on a structure. The simplest and most extensively used analysis assumes the acceleration of all parts of the structure to conform to the acceleration of the ground. The transformation of accelerations to forces is made by applying Newton's Law: Force is proportional to mass times acceleration. This assumption requires a structure of infinite rigidity and fails to consider the elastic characteristics of real structures. The case of the Nippon Kogyo Bank is cited to illustrate the inadequacy of such a simple assumption; the building was practically undamaged although the estimated ground acceleration was five times the acceleration corresponding to the design lateral force.

Dynamic analytical methods based on the elastic theory of deformations of a uniform cantilever have been proposed by several earlier investigators (56)

(59). Lacking accurate ground motion records, it was impractical to complete these analyses rigorously. The assumption of simple harmonic ground motion was generally made, and the results reflected the steady-state condition of vibration. The conclusions thus established were of limited value, for sinusoidal ground motion does not occur in the destructive zone of an earthquake; nor can steady-state response of the structure be established.

In the absence of sustained harmonic vibrations, which is apparent from the El Centro record, more elaborate dynamic theories are necessary. These are based on the transient response of elastic structures to the irregular ground motion that actually occurs and which has been recorded accurately for a few earthquakes, starting with that at Long Beach, in 1933.

Dynamic Approach to Earthquake Forces

The behavior of structures in earthquakes has generally been recognized as a dynamic vibration phenomenon of a transient nature. Although rigorous solutions are possible for particular ground motions applied to particular structures, these solutions are too involved and of too limited significance to be of direct practical value to the structural engineer. The more rigorous methods, however, should be encouraged to guide the thinking toward less rigorous but more practicable methods.

It is the purpose herein to outline an approach to earthquake forces which, although not new in its basic concept, has been extended to a rational dynamic method for establishing the design forces on a structure. The method involves: first, the determination of the total lateral force or the base shear transmitted into the structure from the ground; and, second, the distribution of that shear as equivalent forces applied to the structure.

Determination of Base Shear.—The base shear for any structure subjected to ground motion is the sum of components each of which corresponds to a mode of vibration. Each mode may be represented by a simple one-mass system as regards the base shear for that mode. Detailed analysis indicates, however, that for practical application only the first or fundamental mode need be considered.

Mathematically, the response of a simple, undamped one-mass system to an earthquake acceleration pattern, as measured by the base shear at a particular time, t, is given by the formula:

$$V_t = \frac{-Wk}{g} \int_0^t f(z) \sin k\,(t - z)\, dz \qquad (1)$$

in which: V_t is the base shear, in pounds; W is the weight, in pounds; k is equal to $2\pi/T$; T is the period of the system, in seconds; g is the acceleration of gravity equal to 32.2 ft per sec per sec; t is the time, in seconds; z is the variable of integration of the nature of time; and $f(z)$ is the applied acceleration function of time. The base shear V_t will vary in an irregular manner with time, depending on the irregular nature of the acceleration pattern, and will have both positive and negative values. However, for design purposes the important values of V_t are the maximum. It is convenient to determine the envelope $\bar{V}_t$ that will

contain V_t, as follows:

$$V_t = \frac{W k}{g} (X^2 + Y^2)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In Eq. 2

$$X = \int_0^t f(z) \sin (k z)\, dz \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3a)$$

and

$$Y = \int_0^t f(z) \cos (k z)\, dz \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3b)$$

The maximum value of the envelope, V, is the absolute maximum base shear that will develop in the one-mass system of period T during the particular earthquake being analyzed. It is called the spectral value of V_t, and if $V = \frac{W \bar{a}}{g}$, then $\bar{a}$ is the spectral acceleration, or the spectral intensity of the earthquake for the period T. If a number of values of $\bar{a}$ are determined for varying periods, T, and if $\bar{a}$ is plotted against T, the resulting curve is the acceleration spectrum and is characteristic of the particular earthquake.

Although the El Centro record exhibits an irregular pattern and lacks any semblance of a simple harmonic motion and although the spread of acceleration periods is wide, there are definitely dominating periods between 0.1 sec and 0.3 sec. Other minor and moderate earthquakes recorded seem to have peak acceleration periods in this same general range. The accelerations produced by explosive shocks, which simulate minor earthquakes, also exhibit this short period predominance. It might be expected, therefore, that a one-mass system subjected to actual earthquake acceleration patterns would build up a greater response or resonance effect when its natural period, T, is in the range of the dominant earthquake acceleration periods; and it might also be expected that when the period, T, is removed from the dominant range of the earthquake, the response would decrease. Available data confirm this view.

The response of an undamped one-mass system can be determined analytically or experimentally as solutions of Eqs. 1 or 2. Experimentally, a model such as a torsion pendulum can be subjected to an angular disturbance of the same pattern as the recorded ground motion, and the resulting maximum angular response is the spectral acceleration for the particular period of the torsion pendulum. This is directly proportional to the maximum base shear, V. By a series of such tests, the acceleration spectrum may be obtained, which is indicative of the response or the maximum base shear transmitted from the ground into one-mass systems of varying period, T.

M. A. Biot, using this model method (20) (43) (63) determined the spectra of a number of earthquake patterns, including the Helena aftershocks (1935) and those recorded at Ferndale (Calif.) on medium intensity shocks (1935, 1937, and 1938). From these recordings, an average response curve was obtained which he calls a "standard acceleration spectrum."

Calculations based upon the El Centro record, made at the Watson Scientific Computing Laboratory, New York, N. Y., on the recommendation and under

the direction of E. C. Robison (33), produced similar results. Plotted to a qualitative scale in which the maximum response or base shear is represented by 100, Mr. Biot's standard acceleration spectrum and Mr. Robison's El Centro data are shown in Fig. 5. The two curves have remarkably similar shapes, with peaks at 0.2 sec. and 0.25 sec period, respectively.

Following the Biot Curve, if a one-mass system of 0.2 sec natural period develops a base shear of 100 kips, a one-mass structure of the same weight but with a natural period of 0.73 sec would develop a base shear of only 30 kips under the influence of the same earthquake pattern.

A lateral force coefficient, C, may be defined by the formula:

$$V = C\,W \qquad (4)$$

If W is constant, the lateral force coefficient C is directly proportional to V, the maximum base shear. In effect then Fig. 5 also represents the variation of the

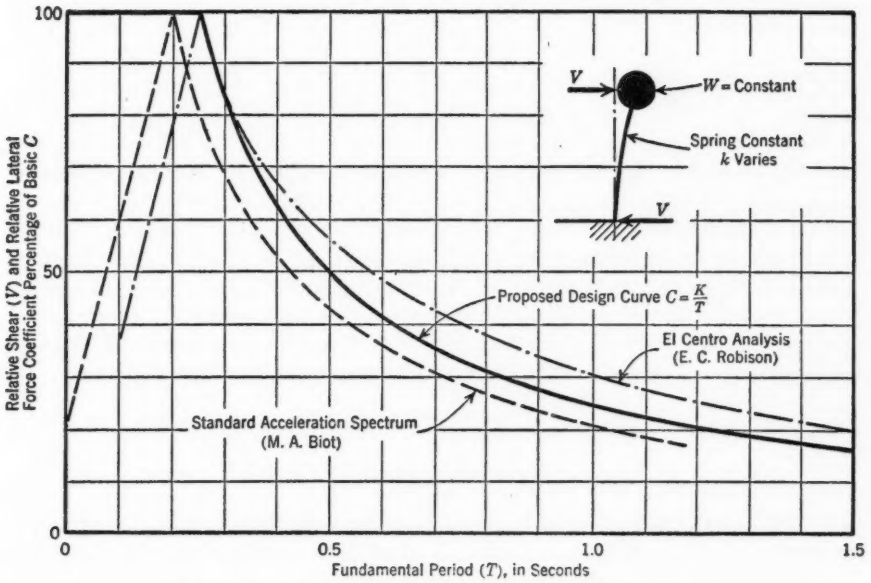


FIG. 5.—RESPONSE OF SIMPLIFIED STRUCTURES TO EARTHQUAKES

lateral force coefficient, C, with the period of the one-mass system; and since the one-mass system represents the real structure in its response, these conclusions apply equally well to the real structure.

The calculated quantitative responses of the idealized one-mass systems are considerably higher than experience indicates is necessary in the design of real structures. Mr. Biot's standard acceleration spectrum has a maximum value of 1.0 g at $T = 0.2$ sec. All the effects of such factors as damping, tilting, and slipping in real structures tend to reduce the shear transmitted from the ground. The net result is that the spectra must be treated as qualitative curves and the assignment of quantitative coefficients must be based on the damage experience

in earthquakes, with special reference to structures of conventional design which have withstood an earthquake of major proportions without significant damage.

In practical application, it is inadvisable to reduce the lateral force coefficient for periods less than that corresponding to the maximum coefficient. Therefore, it is proposed to require design for the maximum coefficient for periods from 0 to 0.25 sec. For periods higher than $T = 0.25$ sec, it is proposed to reduce the coefficient along a curve $C = K/T$ in which: K is a constant depending on the maximum coefficient selected; and T is the natural period of the structure in the fundamental mode. This qualitative design curve is shown in Fig. 5 and falls between the Biot curve and the Robison curve. A minimum coefficient should also be established which will apply for all periods higher than that for which $C = K/T = C_{\min}$. The quantitative maximum and minimum coefficients and the corresponding value of K will be discussed in detail after other aspects of the problem are considered. For the present it is sufficient to note that the base shear has been established by two formulas, Eq. 4 and

$$C = K/T \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Distribution of Base Shear as Applied Forces.—The base shear V having been determined, the distribution of that shear as a number of applied lateral forces on the structure becomes the second part of this approach to earthquake forces. For this consideration, two types of simplified structures are analyzed, both being cantilevers of uniform weight per unit of height: Case I for which the deflection is entirely due to bending; and Case II for which the deflection is entirely due to shear, or story-to-story bending combined with shear. Both these simplified structures are capable of vibrating in an infinite number of modes. For any particular mode, the deflection curve also represents the lateral force curve or the distribution of the base shear as applied lateral forces up through the structure. The deflections are given by the formulas (4*a*): Case I—

$$y = B\,(\cos k\,x - \cosh k\,x) + D\,(\sin k\,x - \sinh k\,x) \quad \ldots\ldots\ldots (6a)$$

Case II—

$$y = \bar{a} \sin m\,x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6b)$$

in which: y is the deflection at height x; x is the height above the base; k and m are constants depending upon the mode of vibration and physical properties of the structure; and $\bar{a}$, B, and D are constants depending on end conditions. For Case I the lateral force is proportional to the fourth derivative of y with respect to x, $\left(\frac{d^4y}{dx^4}\right)$, which has exactly the same form as the equation for y. For Case II the lateral force is proportional to the second derivative of y with respect to x, $\left(\frac{d^2y}{dx^2}\right)$, which again has exactly the same form as the equation for y.

The distribution of a unit base shear ($V = 1$) applied to a structure of unit height ($H = 1$), vibrating in the fundamental mode is shown in Fig. 6(*a*) for Case I bending deflection, Case II shear deflection, and Case III conventional static lateral load proportional to weight distribution. For most structures, the distribution of the unit base shear falls between the curves for Cases I and

II as limiting conditions of deflection. For slender structures, it will be in accord with Case I curve; for squatty structures, it will be in accord with Case II curve. The Case III curve is an empirical criterion for establishing a lateral force, adequate in some cases but not technically logical.

If the deflection δ at the top of the structure, vibrating in the fundamental mode, is half due to shear and half due to moment, this combination is shown in Fig. 6(*b*), the mean between w_M and w_V, for $\delta_M = \delta_V$ being

$$w' = \tfrac{1}{2}(w_M + w_V) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

This curve approximates very closely a triangular loading, zero at the base and twice the average lateral load at the top. For structures approximating uniform weight per unit height and for which not more than half the deflection results from moment, the triangular lateral load gives a conservative basis for the distribution of the base shear. For structures in which moment deflection predominates, the triangular loading is still adequate although greater refinement can be used by distributing the base shear directly in accordance with

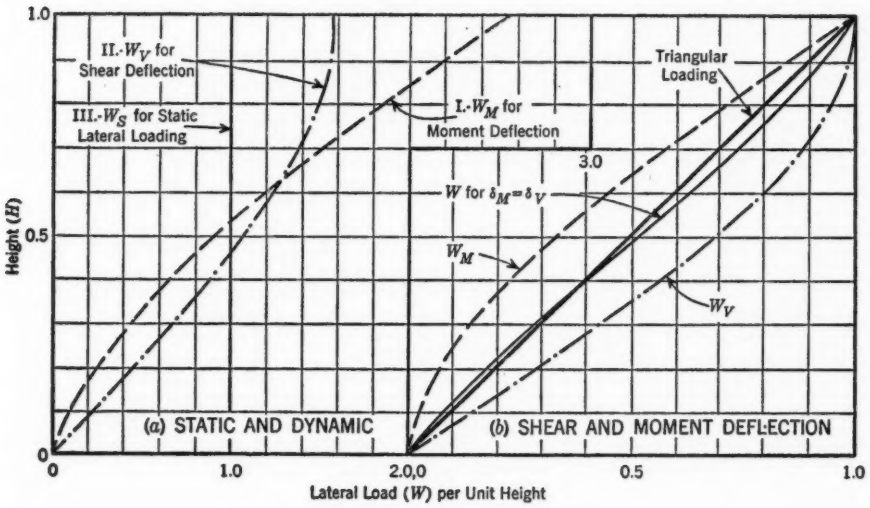


FIG. 6.—LATERAL LOAD DISTRIBUTION

curve I of Fig. 6(*a*). A more general form of distribution, which assumes linear deflection but which permits the variable distribution of the masses or weights of the structure, is given by the formula

$$F_x = V\left[\frac{w_x h_x}{\sum (W h)}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For uniform weight per unit height, assumed in developing the curves in Fig. 6, Eq. 8 reverts to the triangular lateral loading.

The triangular loading or the formula for F_x (Eq. 8) determines the vertical distribution of the base shear. To complete the application of these forces, they

must be distributed over the area of the structure at level x. This should be done in accordance with the mass distribution on that level.

Shear Curves.—The integration of the lateral force curves of Fig. 6 for a unit base shear results in the shear curves (numbered 1, 2, 3, 4) in Fig. 7. The triangular loading shear curve, curve 1 is closer to the shear curve for shear deflection, curve 2, than it is to the shear curve for moment deflection, curve 4. This is reasonable for the average structure. Tests made by J. A. Blume, M. ASCE, on the Alexander Building in San Francisco (22), show that this rather

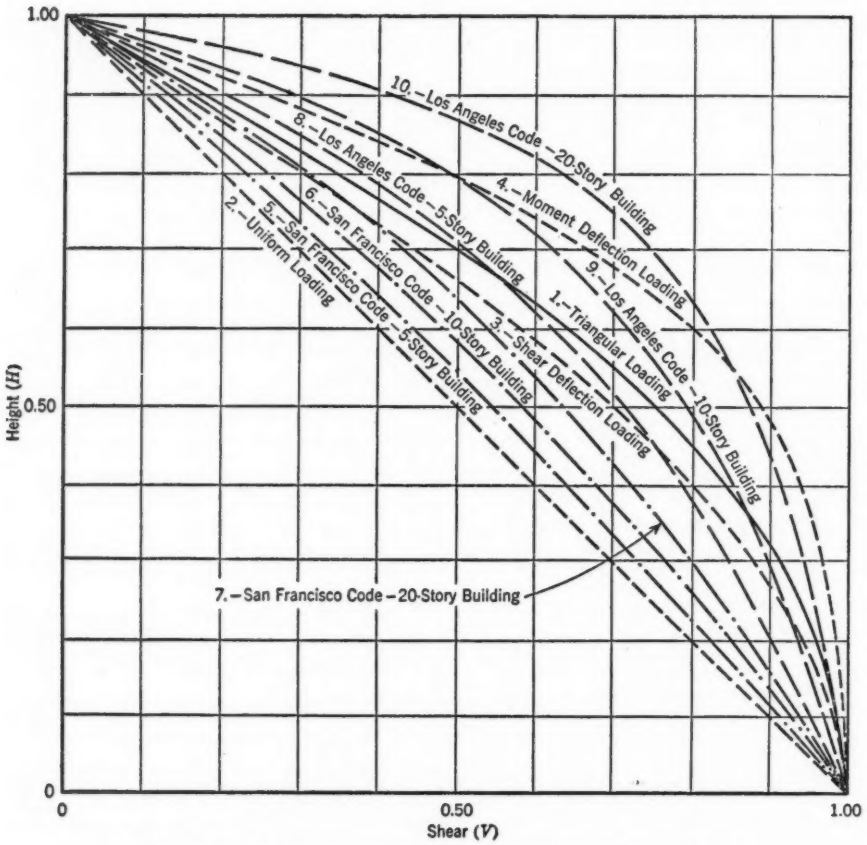


FIG. 7.—SHEAR DISTRIBUTION COMPARED TO EXISTING CODE REQUIREMENTS

tall, slender structure (60 ft by 68 ft by 196 ft high) deflects roughly half in shear and half in moment, excluding the effects of foundation rotation. Foundation rotation also produces a triangular lateral force distribution. Since most buildings are relatively squattier, the proposed triangular lateral loading or the linear deflection curve is generally on the safe side.

A comparison of the shape of the triangular loading shear curve with the shapes of the shear curves for five-story, ten-story, and twenty-story buildings in accordance with the current codes in San Francisco and Los Angeles is shown

in Fig. 7. It appears that the San Francisco code, irrespective of the actual quantitative lateral force coefficients, tends to reduce the shears too rapidly up through the structure. All three curves for five-story, ten-story, and twenty-story buildings fall below the shear curve for shear deflection only, which curve should be the lower limit of base shear distribution. On the other hand, the Los Angeles code seems to carry the shears too high, particularly for the taller buildings. Perhaps concern for the effects of higher modes of vibration prompted this standard. However, if the shape of the fundamental mode shear curve is accepted as an adequate envelope to contain the effects of higher modes, then the upper-story coefficients in the Los Angeles code are too high with respect to the over-all or base shear coefficient.

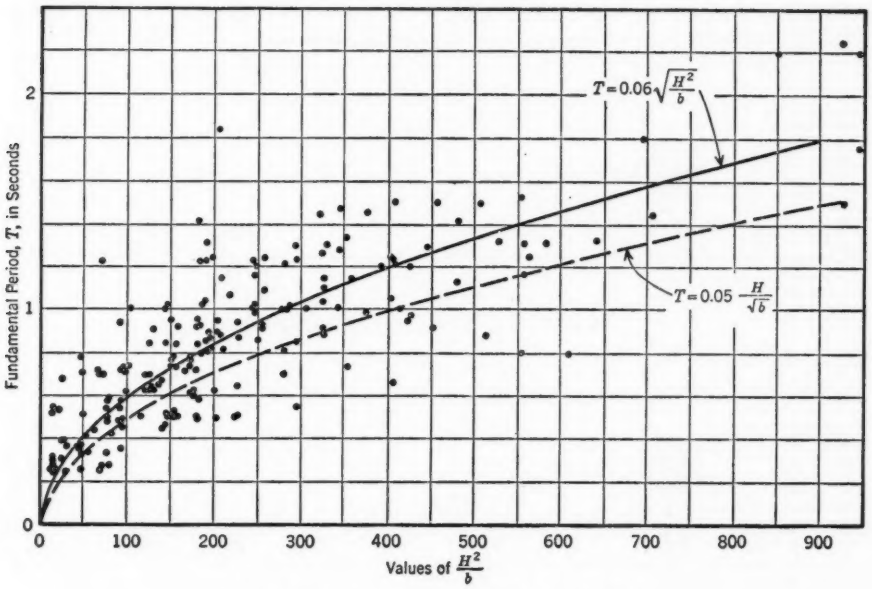


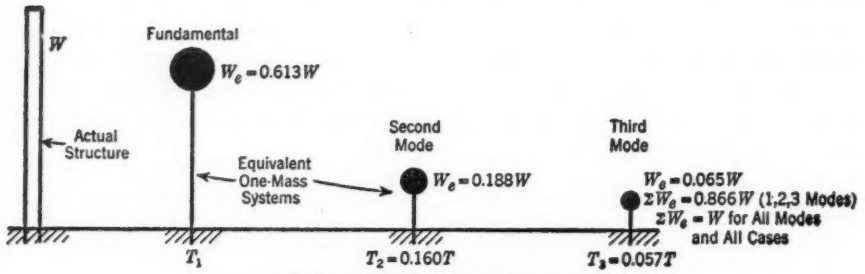
FIG. 8.—FUNDAMENTAL PERIOD OF BUILDINGS

Determination of Period.—It has been shown (Eqs. 4 and 5) that the base shear can be calculated from $V = C\,W$ and that $C = K/T$. In order to determine C, some reasonable means must be devised for predetermining the period T of a structure. It will have to be known (approximately, at least) from the basic dimensions and possibly some of the other physical characteristics of the structure. Comparison with actual similar structures of known period is one approach to this determination. The USCGS has made many measurements of the period for buildings and other structures. Up to July 1, 1949, approximately 1,600 vibration observations had been made in 430 buildings, 150 observations on 42 elevated water tanks, 250 special observations, as well as more than 600 ground vibration measurements.

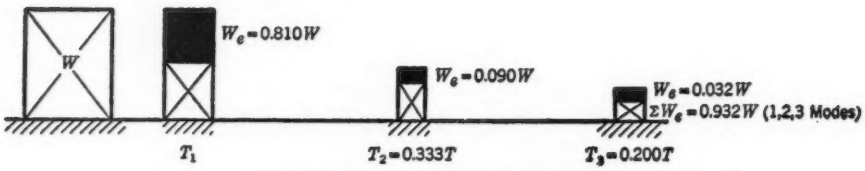
Plotting representative but random building periods observed against H^2/b (the height H squared over the width b in the direction considered), Fig. 8

was obtained. The correlation is not entirely satisfactory, but it is the best of several other simple functions tried. From Fig. 8, an average curve is

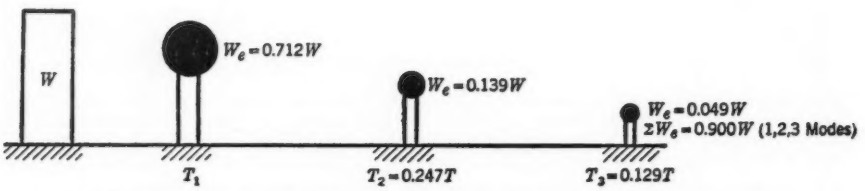
$$T = 0.06\sqrt{\frac{H^2}{b}} = 0.06\frac{H}{b^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9a)$$



(a) CASE I MOMENT DEFLECTION ONLY



(b) CASE II SHEAR DEFLECTION ONLY



(c) CASE III SHEAR AND MOMENT DEFLECTION (AVERAGE OF CASES 1 AND 2)

FIG. 9.—EQUIVALENT ONE-MASS SYSTEMS

However, to be more conservative, it is recommended that the period be determined from

$$T = 0.05\sqrt{\frac{H^2}{b}} = 0.05\frac{H}{b^{\frac{1}{2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9b)$$

—unless more direct and more accurate determinations of period are available. Of the points plotted in Fig. 8, 80% fall above (on the conservative side) of the recommended curve, 10% are subject to a possible error of less than 25% on the unsafe side, and the remaining 10% are in error between 25% and 40%. From this standpoint, the simple, easily-applied formula (Eq. 9*b*) is reasonable for

the present. Further research should be undertaken to confine the period predetermination within closer limits without sacrifice of simplicity.

Effect of Higher Modes.—Equivalent one-mass systems can be established for complex structures vibrating in the fundamental mode, and also for higher modes. In each of these one-mass systems the response (as measured by the base shear transmitted from the ground) corresponds to the base shear transmitted from the ground into the real structure in the particular mode of vibration being considered. In each case, the period of the one-mass system corresponds to the period of the complex structure. If the shape of the fundamental shear curve constitutes a good envelope to contain the shear curves for the higher modes, it is reasonable to design for the fundamental mode and thereby take into account adequately the effects of higher modes.

Milton Ludwig (32) has determined ratios of the weight of equivalent one-mass systems, W_e, to the total weight, W, of structures of uniform weight per unit height, for the fundamental and higher modes of vibration, for the case of deflection due entirely to bending, and for the case of deflection due entirely to shear (see Table 2(*a*)). Diagrammatically, this is shown in Fig. 9. These two

TABLE 2.—Relative Base Shears in Fundamental Second and Third Modes of Vibration

Mode	(*a*) Ratios W_e/W			(*b*) Ratios T/T_f			(*c*) Relative Base Shears	
	Case I Bending	Case II Shear	Case III Average	Case I Bending	Case II Shear	Case III Average	Computation	V
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Fundamental	0.613	0.810	0.712	1.000	1.000	1.000	$V_1=0.247\times0.712\ W=0.176\ W$	
Second	0.188	0.090	0.139	0.160	0.333	0.247	$V_2=1.00\times0.139\ W=0.139\ W$	
Third	0.065	0.032	0.049	0.057	0.200	0.129	$V_3=1.00\times0.049\ W=0.049\ W$	

cases correspond exactly to the cases considered under the distribution of the base shear as equivalent applied forces. Case III is an average of the equivalent weights W_e of Cases I and II.

The relationship between the fundamental and higher mode periods for these same cases has been established as shown in Table 2(*b*). Averaging the ratios for Cases I and II is perhaps a rather arbitrary way to arrive at some in-between value that represents deflection due to both bending and shear. However, the observed periods in the Alexander Building, for which the deflection was determined to be approximately half due to moment and half due to shear, are in the ratio of 1.000 for the fundamental mode, 0.252 for the second mode, and 0.132 for the third mode, checking very closely the average values for Case III.

If the response of the equivalent one-mass systems follows Fig. 5, the relative influence of the higher modes—particularly the second mode— will be greatest in structures of fundamental period 1.03 sec, second mode period 0.250 sec, and third mode period 0.130 sec. This establishes maximum coefficients for the second and third modes and a low coefficient for the fundamental mode. The relative base shears developed in these three modes will be as shown in

Table 2(*c*). These base shears and their distribution are plotted over the height of the structure, using the triangular distribution for the fundamental mode, to obtain Fig. 10. The second mode distribution corresponds to the average of the shear and moment deflection curves in this mode, and the third mode distribution corresponds to the shear deflection curve.

The higher modes, particularly the second mode, can have a very important part in shear and moment and stress distribution in the height of the structure (see Fig. 10); but the significant point is that the shape of the fundamental mode shear curve supplies a reasonable envelope to contain the effect of the higher modes. If a structure has been designed for a practical base shear coefficient and if this base shear is distributed in the triangular fashion, the resulting structure will have reasonably well-balanced strength to resist the

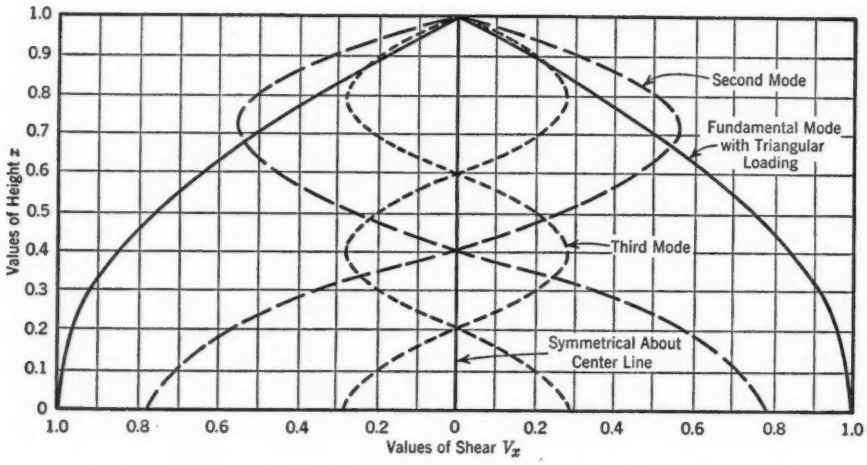


FIG. 10.—RELATIVE SHEARS FOR THE FUNDAMENTAL, THE SECOND, AND THE THIRD MODES

combination of shears of various modes to which the structure will actually be subjected in an earthquake. A factor that has not been considered in Fig. 10 is damping, which reduces the response of the fast-moving higher modes more than it does the fundamental. The effect of higher modes is less for buildings and structures for which the shear deflection predominates, or conversely, the fundamental mode is even more important.

Experiments conducted with a dynamic building model on a shaking table at Stanford University (42) indicated that shears for the second and third modes of vibration were considerably greater than for the fundamental mode for a uniformly rigid model. These experiments have made a notable contribution to our over-all understanding of the dynamic effects of earthquakes. However, the simplifying assumptions which had to be made in the model experiments magnify the influence of the higher modes. A constant duration of 2.2 sec was employed and also a constant initial velocity of the ground or table. The ensuing ground motion was a damped harmonic vibration of prearranged period. With the constant duration, if the fundamental mode represented 1.63 cycles of

applied harmonic vibration, the second mode was 5.61 cycles, and the third mode, 11.82 cycles for the model of the Alexander Building, San Francisco. While there is a dominance of higher frequencies in an actual earthquake motion, it is doubtful whether the ratios of cycles would ever reach 1.63: 5.61: 11.82 in any 2.2 sec interval.

Further experiments with simplified models, including equivalent one-mass systems, would be helpful in establishing or corroborating the relative importance of fundamental and higher modes. The models should be subjected to actual recorded earthquake motion and provision should be made for the effects of damping.

Establishment of Coefficient C

Determination of W.—Before establishing the quantitative coefficient C which should apply to buildings and to other structures, two other items should be considered which can be defined independently but which, when defined, influence the selection of the coefficients. One is the determination of the weight of the structure, W, to which the coefficient C is to be applied to arrive at the base shear V (Eq. 4). The other is the consideration of the allowable increase in stress for the short-time loading condition prevailing in an earthquake.

The basic principle is universally accepted that W should be the probable load existent at the time of the earthquake—the load contributing to the masses to be accelerated. The entire dead load, and any live load that might reasonably be expected, should be included. The structural elements of a building are designed to carry any vertical loads to which, at any time, they may be subjected. It is usually required that office floors be checked for concentrated loads; but it is not logical to assume such loads over the entire office floor for determining seismic forces.

A comprehensive study of the actual live loading in many types of buildings was made by the Department of Commerce Building Code Committee (28) in 1923. The actual average live load in office buildings, hospitals, hotels, and residences was approximately 10 lb per sq ft. Retail stores averaged about 25 lb per sq ft. Public corridors may be loaded up to 50 lb per sq ft or even 75 lb per sq ft, but in such cases the adjoining rooms, in all probability, would not be occupied. Storage areas, warehouses, etc., naturally vary greatly in their loading, dependent on such factors as the materials and method of storing. However, due to the necessary aisles, passageways, and unloaded areas, an average live load in warehouses of 50% of the design live load is adequate for seismic design. In view of these considerations, the load W to be used in lateral design for buildings is defined as the total weight of the structure above the base including

> * * * all dead load plus 50% of the design live load for storage and warehouse floors, 25% of the design live load for all other floors, and no live load for roofs.

For structures other than buildings, W is defined as dead load plus normal operating live loads.

Allowable Stress Increase.—Practically every current code allows one-third increase in normal design stresses for combined stresses from lateral forces and vertical loads, provided the resulting section is not less than that required for the dead loads and live loads alone. Also, for members carrying lateral force stresses only, the permissible stress is one third greater than normal.

The one-time proposed California Building Code which was sponsored by the State Chamber of Commerce (29) included provisions for stress increase of 100% for combinations including earthquake loading, except that for columns the maximum increase permitted was 75%.

There is a sound basis for these higher increases in allowable working stresses under earthquake conditions. They provide for a better balanced design. Members which normally carry full stress under vertical loading and members which carry virtually no stress under vertical loading approach failure together as extreme lateral forces are applied. The moderate one-third increase in allowable working stress results in unbalance such that certain members are stronger than others. The weakest members are those which normally carry no stress and the strongest are those which are fully stressed under vertical loading. This is a well-known fact and can be demonstrated mathematically. Further study should be made of the interrelation between the allowable stress increase, the balanced character of the design, and the adopted seismic coefficients.

Pending such study, however, the Joint Committee has retained the presently accepted one-third allowable stress increase and has set the lateral force coefficients on this basis.

Quantitative Coefficient C.—In the present state of knowledge, the choice of the maximum and minimum lateral force coefficients must be based on engineering judgment and experience. Correlated factors, such as the definition of the weight W and the allowable increase in stress for combined lateral and vertical loading, have been discussed and the coefficients chosen are necessarily related to these factors.

To arrive at these coefficients, the Joint Committee has considered in detail the following basic points:

1. It is the province of the building code to specify design and construction requirements which will result in structures safe against major structural damage and loss of life in the event of precedented earthquakes.

2. The extensive review of earthquake history and damage, and the calculated resistance of buildings and structures which have successfully gone through a major shock, have not revealed major structural damage in any California building that had a lateral resistance corresponding to C = 0.02 or greater based on current design practices. In fact, many buildings with less lateral resistance than $C = 0.02$ have successfully withstood major earthquakes. The Flood Building with a calculated resistance less than $C = 0.006$ has been mentioned. Other buildings designed for wind forces, again corresponding to low earthquake coefficient forces by current design practices, also withstood major earthquakes successfully.

3. Damage and loss of life that did occur in such buildings was caused by inadequate anchorage of elements such as parapets, cornices, filler walls, parti-

tions, stair and elevator walls, stair treads and stringers, and in appurtenant structures such as roof tanks. The anchorage of such items requires special consideration.

4. An unnecessarily high coefficient for slender buildings requires a type of bracing that is inherently very rigid, and this rigidity increases the probability of damage.

The Joint Committee was unanimous in its selection of $C = 0.06$ as the maximum base coefficient for buildings and $C = 0.02$ as the minimum base coefficient. Hence, K in the formula $C = K/T$ must be 0.015, as shown graphically in Fig. 11.

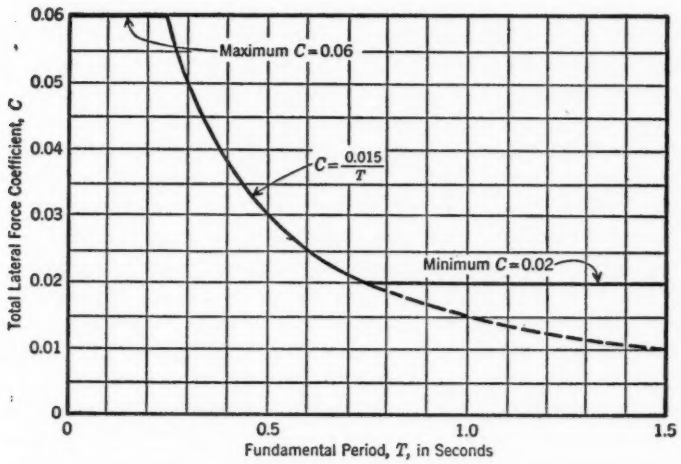


FIG. 11.—LATERAL FORCE COEFFICIENTS FOR BUILDINGS

SPECIAL CONSIDERATIONS

Special conditions or considerations with respect to earthquake forces on buildings include: Load-carrying capacity of the soil; moment-resisting frames; overturning moment; rigidities and torsional moments; setbacks and towers; lateral forces on parts of structures; and combined axial and bending stresses in columns.

Load-Carrying Capacity of the Soil.—It has been generally (although not consistently) observed that damage in earthquakes has been more severe on alluvial or filled ground than on firm ground or rock. Records show greater movement and larger accelerations in the softer ground. However, it has also been observed that certain rigid buildings fared better on the soft ground than they did on the firmer soils. This was the case in Tokyo. Kyoji Suyihiro and Mishio Ishimoto (64) stated that "* * * there is ample evidence to show that rigidly constructed buildings withstood the earthquake better downtown than uptown," although the relative number of buildings destroyed proved conclusively that the earth motion was more severe downtown where the soil was alluvial and more unstable than uptown. They further state:

> "* * * the view that the foundations of rigidly constructed buildings move in just the same manner as the surrounding ground seems to us to be erroneous. If the surrounding ground were of hard rock and the foundations rigidly built upon it, the foundations would follow the ground. However, for buildings standing on soft alluvial ground like downtown Tokyo, the condition should be different. In this case, the rigidity of the buildings being far greater than that of the ground on which they stand, the buildings should behave like pebbles placed on gelatin and there is possibility that they actually rock as a whole upon the soft ground."

It has been found that the rotation of a building on its base, in vibrating, has a very considerable effect in lengthening the natural periods of vibration of the building, especially the fundamental period. This rotation is of greater effect on a rigid building than on a flexible one. The rotation of a building on its foundation has a cushioning effect, which decreases the dynamic shears and moments throughout the building in all modes.

The observed greater damage in alluvial ground is apparently due to other factors as much as or more than to the soft ground itself. Consideration must be given to the types of construction on the two areas—the soft soil and the firm soil—and it may be found that inferior construction occurred in the poorer soil area. Again, settlement damage that has occurred in alluvial soils in earthquakes cannot be corrected by increasing the lateral force factors.

Several codes already recognize that current knowledge is insufficient to justify different lateral force factors for different soil conditions. The Joint Committee took this same view and established only one set of coefficients.

Moment-Resisting Frames.—The history of earthquake damage indicates that buildings with moment-resisting frames, designed usually for wind without earthquake considerations, have had a very good record. A frame encased in more rigid walls can resist only a very small part of the lateral forces to which a building is subjected. If cracks develop in the more rigid walls, the frame comes into action and provides the structural support for the building "clothing." For earthquakes of long duration, this is especially important.

Recognizing this experience, provisions have been made to allow for the added safety feature of moment-resisting frames, provided that such frames have a resistance not less than 25% of the total lateral force requirement. Any resistance built into the frame may be subtracted from the total lateral force requirement to arrive at the resistance to be carried by other elements; but the building as a whole (that is, in accordance with rigidities) must be required to resist a lateral load not less than that given by $C = 0.02$.

For buildings higher than 135 ft, a moment-resisting frame is made obligatory. This frame must be designed for a lateral base shear corresponding to $C = 0.01$, distributed in accordance with Eq. 8, or the prescribed wind force, whichever is the greater.

Overturning Moment.—In recent years the provisions to guard against the overturning of a structure as a whole, or the overturning of lateral force carrying elements of the structure, have varied all the way from no provision to full provision for the overturning effects of the design lateral forces about a fixed base. Without doubt, earthquakes induce moments in buildings; that is, the type of moment by which the floors rotate. On the other hand, there is general

agreement that the requirement for full overturning moment effects on a tall, slender structure is more severe than can be justified by any research to date. This type of structure will tend to vibrate in combinations of higher modes with the fundamental. The full design shears of the fundamental mode (which, as explained, is taken to be the envelope of shear requirements) do not infer that the corresponding overturning moments are realized. The vibrations in the higher modes will cause reversals, adding to and subtracting from each other and the fundamental vibration. Down a certain distance from the top, it is conceivable that the full overturning effect might be realized; but not all the way down—not for the tall slender structures, at least.

The Joint Committee recommends that these overturning moments be calculated for the design lateral forces for the top ten stories of buildings, or for the top 120 ft of other structures, for the full cantilever effect and that moments be considered constant from that point to the foundations. The full overturning effect of the stipulated wind forces must be provided for. Furthermore, the over-all structure and its foundation must have a dead-load resisting moment not less than one and one half times the overturning moment due to these wind forces.

Rigidities and Torsional Moments.—The provision that the forces be distributed in accordance with rigidities requires the designer to distribute the specified lateral loads properly. Rigidities of both horizontal diaphragms and vertical elements carrying lateral load must be considered. Recognition of horizontal torsional moment follows from rigidities. If the rigidities of the elements on any horizontal plane are such as to produce a variation between the center of gravity of the loads and the center of rigidity of the resisting elements, then provision must be made for the twisting or torsional effect induced by the lateral loads on the structure.

Setbacks and Towers on Buildings.—Setbacks and towers on buildings affect the period T. After consideration of the effect of setbacks on the period, both from the standpoint of shear and moment deflections, the Joint Committee concluded that the best solution is to require the calculation of an average height H, found by projecting an elevation of the building on a plane parallel to the direction considered and dividing the area of this projected elevation by the over-all width b. Thereafter, the building is treated as a building without setback. For tower area greater than 25% of the building area below, this method gives reasonable results. For smaller areas, the shears, expressed as a percentage of the tower weight, increase very abruptly. Consequently, an optional method is introduced permitting the designer to consider small towers to be buildings on top of buildings, each designed independently for its own height, width, and weight, but with the shear from the tower applied as an additional lateral force on the top of the building below.

Lateral Forces on Parts of Structures.—It has been mentioned that considerable damage in earthquakes and the injury and loss of life have been attributed to inadequate ties and lack of attention to details. It has been common practice in seismic design and seismic codes to stipulate larger lateral forces on various parts of a building than on the building as a whole. The Joint Committee is in agreement with the coefficients as they are required by current earthquake codes.

Combined Axial and Bending Stress in Columns.—The point in a member, subject to both axial and bending stresses, at which the combined stress should be checked by the formula—

$$\frac{f_a}{f_{aa}} + \frac{f_b}{f_{ab}} \leqq 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

—needs clarification. Adjacent to the support, the full allowable bending stress should be allowed. Based on its judgment, the Joint Committee has indicated that Eq. 10 needs to be checked at and between the quarter points of the unsupported length of the column and that the allowable bending stress f_{ab} shall be permitted between the quarter point and the support.

Structures Other Than Buildings

For structures other than buildings, the Joint Committee recommends the same general approach as that outlined for buildings. However, due to the generally simpler structures involved, with less damping action and little if any reserve strength such as is usually available in buildings, it is recommended that higher coefficients be used. The maximum C proposed is 0.10, the minimum C is 0.03, and K is 0.025. No formula is given for determining the period T, which must be calculated by recognized methods.

Because of experience with braced tank towers, their lack of reserve strength, the rigidity of bracing, and the susceptibility of the bracing to impact stresses, it is stipulated that these towers be designed for a C-factor of not less than 0.10. Considering the importance of keeping a water supply intact in event of an earthquake, more conservative design appears to be in order.

Illustrative Examples

The application of the recommended code to determine the design earthquake forces in a rectangular five-story building (Fig. 12(*a*)) and in an L-shaped, twelve-story building with a setback (Fig. 12(*b*)) is shown in Table 3. The following are the steps taken in the direction of each principal axis:

a. Determine the period of the structure, T;

b. Using this period, calculate the base shear coefficient, C;

c. Apply this coefficient to the total weight of the building to determine the base shear, V;

d. Tabulate the height to each floor, h_x, and the weight at each floor, w_x, for dead load and proper percentage of live load;

e. Tabulate the product of weight and height for each floor, $w_x h_x$, and sum these products for the entire building to $\sum(w h)$;

f. Determine the ratio $\frac{V}{\sum(w h)}$ by which each product $w_x h_x$ is to be multiplied to determine F_x, the portion of the base shear V to be applied at that floor; and

g. Tabulate the F_x for all floors, and sum these to check with the base shear V.

In Table 3, Cols. 7 and 9, a further step is shown in recording the ratio of the forces F_x to the weight w for each floor. This is not normally required but is shown here to illustrate that the equivalent coefficients in the top stories, as

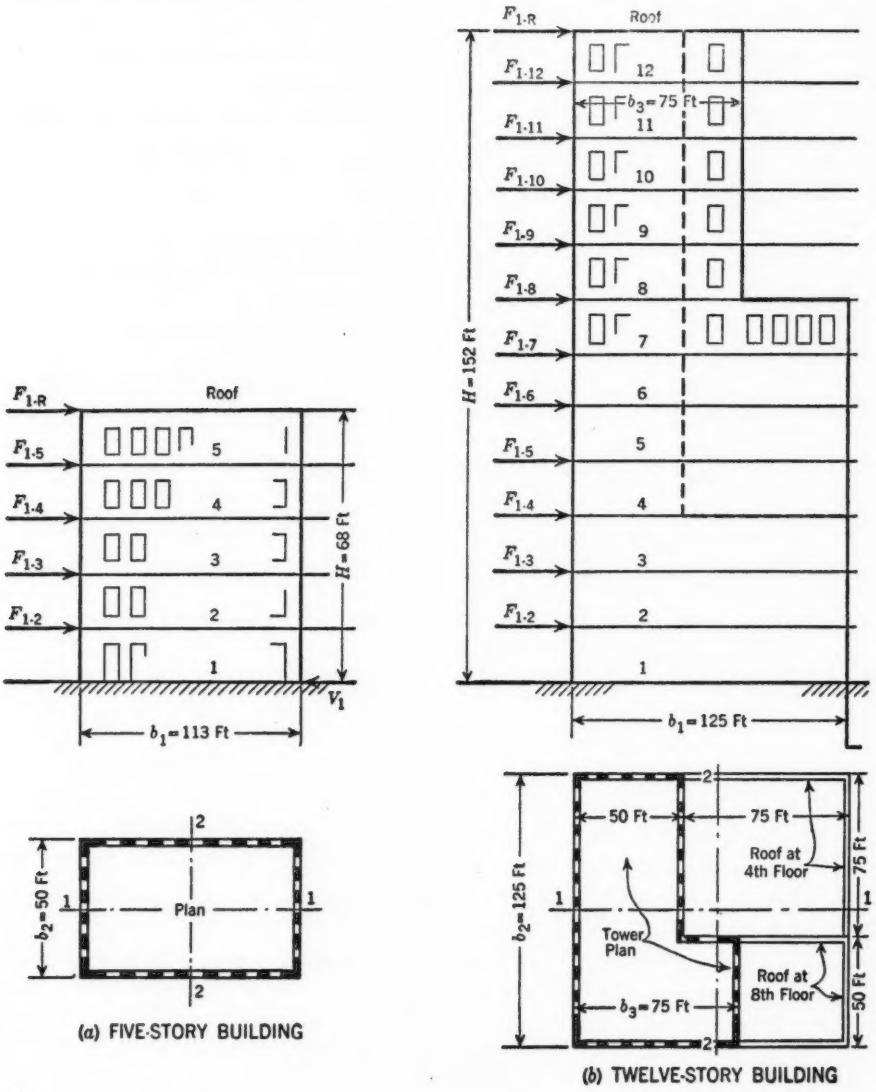


FIG. 12.—BUILDINGS SELECTED FOR ILLUSTRATIVE EXAMPLES (SEE TABLE 3)

defined in current codes, are approximately double the base shear coefficients. Thus in the five-story building, the equivalent top-story coefficients in the two directions are 7.95%, and 5.62% compared to the over-all or base shear coeffi-

cients of 4.68% and 3.31%, respectively. Similarly, in the twelve-story building, the equivalent top-story coefficients are 5.45% and 4.69%, compared with base shear coefficients of 2.62% and 2.21%, respectively. The effect of the setback is to require somewhat higher lateral forces in the upper parts of the struc-

TABLE 3.—COMPUTATIONS FOR ILLUSTRATIVE EXAMPLES FOR 50 BUILDINGS SHOWN IN FIGURE 12.

Floor	Δh (ft)	h (ft)	w (kips)	$w h$[a]	F_1 (kips)	$\frac{F_1}{w}$[b]	F_2 (kips)	$\frac{F_2}{w}$[b]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
(a) Five-Story Building								
R	→	68	500	34	39.7	7.95	28.1	5.62
5	12 →	56	1,000	56	65.4	6.54	46.4	4.64
4	12 →	44	1,000	44	51.3	5.13	36.4	3.64
3	12 →	32	1,000	32	37.3	3.73	26.5	2.65
2	12 →	20	1,000	20	23.3	2.33	16.6	1.66
1	20	0	150	0	0	0	0	0
Totals			4,650 = W	186 = $\Sigma(wh)$	217.0 = V_1	...	154.0 = V_2	...
(b) Twelve-Story Building								
R	→	152	750	114	40.8	5.45	35.2	4.69
12	12 →	140	1,500	210	75.2	5.01	64.9	4.32
11	12 →	128	1,500	192	68.8	4.58	59.3	3.95
10	12 →	116	1,500	174	62.3	4.15	53.7	3.58
9	12 →	104	1,500	156	55.8	3.72	48.2	3.21
8	12 →	92	1,760	161	57.6	3.29	49.7	2.84
7	12 →	80	2,000	160	57.4	2.87	49.4	2.47
6	12 →	68	2,000	136	48.6	2.43	42.0	2.10
5	12 →	56	2,000	112	40.1	2.01	34.6	1.73
4	12 →	44	2,562	113	40.5	1.58	34.9	1.36
3	12 →	32	3,125	100	35.7	1.14	30.9	0.99
2	12 →	20	3,125	62	22.2	0.71	19.2	0.61
1	20	0	300	0	0	0	0	0
Totals			23,612 = W	1,690 = $\Sigma(wh)$	605.0 = V_1	...	522.0 = V_2	...

Computations

(10)

Five-Story Building:

$$T_1 = \frac{0.05\,H}{\sqrt{b_1}} = \frac{0.05 \times 68}{\sqrt{113}} = 0.320$$

$$T_2 = \frac{0.05 \times 68}{\sqrt{50}} = 0.453$$

$$C_1 = \frac{0.015}{T_1} = \frac{0.015}{0.320} = 0.9468 - 0.0468$$

$$C_2 = \frac{0.015}{0.453} = 0.0331$$

$$V_1 = C_1 W = 0.0468 \times 4{,}650 = 217$$

$$V_2 = 0.0331 \times 4{,}650 = 154$$

$$F_{1-x} = \frac{V_1 w_x h_x}{\Sigma(w h)} = \frac{217}{186} w_x h_x = 1.17\, w_x h_x$$

$$F_{2-x} = \frac{154}{186} w_x h_x = 0.828\, w_x h_x$$

Twelve-Story Building:

$$H_1 = \frac{92 \times 125 + 60 \times 75}{125} = 128$$

$$H_2 = H = 152$$

$$T_1 = \frac{0.05\,H_1}{\sqrt{b_1}} = \frac{0.05 \times 128}{\sqrt{125}} = 0.571$$

$$T_2 = \frac{0.05 \times 152}{125} = 0.678$$

$$C_1 = \frac{0.015}{T_1} = \frac{0.015}{0.571} = 0.0262$$

$$C_2 = \frac{0.015}{0.678} = 0.0221$$

$$V_1 = C_1 W = 0.0262 \times 23{,}612 = 605$$

$$V_2 = 0.0221 \times 23{,}612 = 522$$

$$F_{1-x} = \frac{V_1 w_x h_x}{\Sigma(w h)} = \frac{605}{1{,}690} w_x h_x = 0.358\, w_x h_x$$

$$F_{2-x} = \frac{522}{1{,}690} w_x h_x = 0.309\, w_x h_x$$

[a] Thousands of foot-kips or millions of foot-pounds. [b] Percentages.

ture, as measured by the equivalent story coefficients. The top-story coefficients in Fig. 12(*b*) (Table 3(*b*)) are slightly more than double the base shear coefficient, whereas the corresponding ratio for a twelve-story uniform plan building would be slightly less than double.

WIND

Wind forces and their effects have been reported ably by other investigating committees. This Joint Committee reviewed much of the work which has been done in this field in recent years, and concluded that no major changes from the specification of current codes were necessary.

The basic measure of wind pressure on a structure can be approximated for engineering design purposes by the formulas:

$$P = c\,q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and

$$q = 0.00256\,v^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which: v is the design velocity of the wind, in miles per hour; q is the velocity pressure, in pounds per square foot; c is the shape factor of the part or whole of the structure; and P is the design pressure, in pounds per square foot.

The 5-min average design velocity, plus a gust factor, is obtained from records of the United States Weather Bureau. The American Standards Association (ASA) recommends gust factors varying with height above the ground (36). At 30 ft, the factor is 1.5; at 500 ft, 1.3; at 1,000 ft, 1.23; at 1,500 ft and higher, 1.2. After extensive investigations at the University of Michigan, in Ann Arbor, R. H. Sherlock, M. ASCE, recommends (25) a gust factor of 1.385 for low buildings compared with 1.5 by ASA (36). The 1.385 factor corresponds to a 3-sec gust, the minimum time necessary to develop the maximum pressure on a building as a whole.

For San Francisco, 78 years of observation give a maximum 5-min average velocity of 50 miles per hr, 112 ft above the ground. Using the relationship that the wind velocity varies as the one-seventh power of the height, this gives a velocity pressure for this area by Eq. 12. Using the more conservative gust factor of 1.5, the wind pressure is $q = 0.00256 \left[1.5 \times 50 \times \left(\frac{30}{112} \right)^{1/7} \right]^2 = 9.88$ lb per sq ft at 30 ft above ground. This accords with the ASA velocity pressure recommendation for Northern California, which is 9 to 10 lb per sq ft.

The shape factor varies considerably. For structural shapes, such as small angles, channels, squares, H-beams, H-columns, and flat plates, it varies from 1.62 to 2.19 when the wind is free to go all around the member. For buildings and structures of "average" size on the ground, it is generally assumed that the shape factor c is 1.3, consisting of 0.8 pressure on the windward side and 0.5 suction on the leeward side.

For San Francisco conditions, this value would give a design pressure P, the total windward and leeward effect, of $P = 1.3 \times 9.88 = 12.85$ lb per sq ft at an elevation of 30 ft. Based on the Sherlock gust factor of 1.385, this design pressure would be less; that is, 10.95 lb per sq ft instead of 12.85 lb per sq ft. Also for heights less than 30 ft above ground, the pressure would be further reduced, so that a building 30 ft high with a design pressure at the top of 12.85 lb per sq ft would have an average lateral wind force of less than 10 lb per sq ft.

Thus, there is considerable evidence to support a reduction of the design pressure for low buildings and structures. From the standpoint of safety alone this could be done. However, the Joint Committee felt that to provide reasonable rigidity against deflection, current wind pressures specified in this area should be continued. These are 15 lb per sq ft on structures up to 60 ft high and 20 lb per sq ft above this point. For purposes of design of the structure as a whole, these pressures can be applied entirely on the windward side because lateral forces on parts of the structure include an inward or outward force on walls not less than 20% of the wall weight nor less than 15 lb per sq ft.

For small roof structures, subject to higher wind velocities and higher gust factors, 30 lb per sq ft wind pressure is specified. Also on open-frame structures for which the shape factor is higher and for which the leeward side framing is exposed to a reduced velocity, 30 lb per sq ft wind pressure on one and one-half times the area of the members in the windward exposure is recommended.

It should be emphasized that, although the lateral forces specified are generally satisfactory and conservative for most of Northern California, there are locations of extreme exposure, fetch, and height which require higher factors. Point Reyes Lighthouse, for example, reported a 5-min average velocity estimated at 84 miles per hr in 1914, with a continuous record from 1911 to 1944. This would correspond to a wind design pressure of 36 lb per sq ft, compared to the 12.85 lb per sq ft for the San Francisco record. This record at Point Reyes, however, is not typical of the other record stations. It would not be in order to penalize a large area by excessive design wind pressures because of a localized condition. Designers should be cautioned, however, to consider the local conditions, wind record, exposure, etc., in determining the design wind forces.

Recommendations for Future Study

In the development of the recommended code, several problems were encountered which require further research either to substantiate the current recommendations, or possibly to modify or refine them. Although they are not all-inclusive, suggestions for future research include:

1. More extensive, accurate records of ground motion, encompassing on a uniform basis not only continental United States but all of the more active seismic regions of the world;
2. Additional response curves for one-mass systems, including the effects of damping, preferably to be made a part of the analysis of the ground motion records and reported therewith;
3. More complete examination of the periods of vibration of buildings and other structures, and the factors affecting these periods, and establishing a more accurate yet practicable criterion for determining these periods for lateral force design purposes;
4. Study of the effects of soil conditions on ground motion and the response of structures, including the effects of rotation in softer soils; and the influence of these factors on the lateral force coefficient C;
5. Additional model and shaking table studies, particularly relating the response of one-mass systems and equivalent one-mass systems to the responses of multimass prototypes;

6. Study of the rigidity criterion for the distribution of design lateral forces for various types of structures and conditions encountered, preferably to be reported as a design practice;

7. Further study of allowable increases in working stresses for short-time lateral loading combined with normal vertical loading to include the basic structural materials and, if increases higher than the current practice of 33% are in order, to consider the effects of these upon the lateral force coefficient C;

8. Establishment of a design criterion for the overturning effects in earthquakes, based upon dynamic considerations;

9. Cataloging and indexing of buildings and other structures which have been designed for seismic forces, with basic structural data, so that in the event of a major earthquake, an early report of damage or lack of damage and its relationship to the design can be compiled;

10. Further study of wind effects on low buildings for California conditions, and consideration of modification of lateral wind pressures for these structures, if this is found warranted; and

11. Study of earthquake effects on bridges that are not covered in this paper because they do not normally come under the jurisdiction of the building code.

Acknowledgments

The Joint Committee on Lateral Forces was appointed April 2, 1948, by the Boards of Directors of the San Francisco Section, ASCE, and the Structural Engineers Association of Northern California. This paper, based on the report of the Joint Committee, is presented under the joint authorship of the members of this committee.

APPENDIX I.—LATERAL FORCE CODE

The following code recommendations are numbered for adaptation to a larger code. For example, if "Lateral Forces" were included as Sect. 8 of the more comprehensive code, Sect. 01 would then become Sect. 801, etc.

Sect. 01.—General Requirements

Every building and other structure, in all its parts and details, shall be designed and constructed to resist and withstand lateral forces herein described from any direction due to wind forces or earthquake forces, whichever are greater. The forces considered shall be not less than those described in Sections 02, 03, and 04. In general, every building or structure shall be so designed that all its elements calculated to resist lateral forces are tied and bonded together.

Sect. 02.—Wind Pressure

(*a*) For purposes of design the wind pressure shall be taken on gross area of the vertical projection of buildings and structures at not less than 15 lb per sq ft for those areas of the building or structure less than 60 ft above ground and at not less than 20 lb per sq ft for areas more than 60 ft above ground. Roofs and their anchorages shall be designed for an uplift force of 10 lb per sq ft.

(*b*) The wind pressure on roof tanks, roof signs, or other exposed roof structures and their supports, shall be taken as not less than 30 lb per sq ft of the gross area of the plane surface, acting in any direction.

(*c*) In calculating the wind pressure on circular tanks, towers, or stacks, these pressures shall be assumed to act on six-tenths of the projected area.

(*d*) On open-framed structures the area used in computing wind pressure shall be one and one-half times the net area of the framing members in the side exposed to the wind using a base pressure of 30 lb per sq ft.

Sect. 03.—Minimum Earthquake Forces for Buildings

(*a*) *Total lateral force.*—Every building shall be designed and constructed to withstand minimum total lateral forces, determined independently in the directions of the principal axis of the building, as given by the formula

$$V = C/W$$

in which: V is the total lateral force or shear at the base; C is the numerical coefficient as given hereafter; and W is the total weight of the building above the base, including dead load plus a percentage of the live load hereinafter specified.

(*b*) *Coefficient C.*—

1. In the building as a whole the coefficient C shall be

$$C = \frac{0.015}{T}$$

in which: T is the fundamental period of vibration of the building in seconds in the direction considered. The required value of C shall not be less than 0.02 nor more than 0.06.

2. Qualified persons may submit technical data to substantiate the period T obtaining for a contemplated building. In the absence of such data, it may be assumed that

$$T = 0.05 \frac{H}{\sqrt{b}}$$

in which: H is the height of the main portion of the building, in feet, measured above the base which shall be the level at which the structure is positively connected to the ground; and b is the width, in feet, of the main portion of the building in the direction considered.

(*c*) *The Weight W.*—The weight W of buildings shall include all dead load plus 50% of the design live load for storage and warehouse floors, 25% of the design live load for all other floors, and no live load for roofs.

(*d*) *Applied Lateral Forces.*—

1. The total lateral force V shall be distributed over the height of the building in accordance with the following formula:

$$F_x = V \frac{w_x h_x}{\sum(w h)}$$

in which: F_x is the lateral force applied to any level x; w_x is the weight at level x; h_x is the height of level x above the base; and $\sum(w\,h)$ is the summation of the products $w\,h$ for the building.

2. At each level x, the force F_x shall be applied over the area of the building, in accordance with the mass distribution on that level.

(*e*) *Lateral Forces on Parts of the Building.*—

1. Parts of buildings and their anchorage shall be designed for lateral forces determined from the formula

$$F_p = C_p\,W_p$$

in which: F_p is the lateral force on the part and in the direction under consideration; W_p is the weight of the part; and C_p is a coefficient selected from Table 4.

TABLE 4.—COEFFICIENTS C_p FOR USE IN SECT. 03(*e*)1

Part	C_p	Comment	Direction
Exterior walls and other nonresisting elements not otherwise classified	0.20	With a minimum of 15 lb per sq ft	Normal to surface of wall
Interior walls and partitions	0.10	With a minimum of 5 lb per sq ft	....
Parapet walls, exterior and interior ornamentations	0.50	....	Any direction horizontally
Towers and tanks, including contents, chimneys, smokestacks, and penthouses when connected to or part of a building	0.20	....	Any direction horizontally

2. The distribution of these forces shall be according to the gravity loads pertaining thereto.

SECT. 04.—MINIMUM EARTHQUAKE FORCES FOR OTHER STRUCTURES

(*a*) All structures other than buildings, except tower-supported structures, shall be designed and constructed to withstand minimum total lateral forces determined as for buildings (Sect. 03) except that C shall be determined by

$$C = \frac{0.025}{T}$$

in which the required value of C shall not be less than 0.03 nor more than 0.10. Qualified persons may submit technical data to substantiate the period T obtaining for a contemplated structure; otherwise T shall be calculated by recognized methods assuming fixed base conditions.

(*b*) In all tower-supported structures, such as elevated tanks, C shall not be less than 0.10.

(*c*) The total lateral force shall be distributed over the height of the structure in the same manner as for buildings, Sect. 03(*d*).

(*d*) The weight W shall include all dead load plus normal operating live load.

(*e*) The provisions of Sect. 03(*e*) shall apply also to parts of structures other than buildings.

Sect. 05.—Distribution of Horizontal Shear

The total horizontal shear at any horizontal plane shall be distributed to the various resisting elements in proportion to rigidities.

Sect. 06.—Horizontal Torsional Moments

Proper provision shall be made for horizontal torsional moments unless the center of rigidity of the resisting structural units is coincident with the center of gravity of the vertical loads in the building or other structure.

Sect. 07.—Provisions Against Overturning

(*a*) The dead load moment of stability of every building or other structure shall not be less than one and one-half times the overturning moment caused by wind pressure.

(*b*) Provision for overturning moment shall be made for the specified earthquake forces in the top ten stories of buildings or the top 120 ft of other structures, and the moments shall be assumed to remain constant from these levels into the foundations.

Sect. 08.—Buildings Having Setbacks

(*a*) Buildings with setbacks shall be designed by the methods previously specified in Sect. 03 except that, for purposes of determining the period T, the coefficient C, and the base shear V, the height H shall be the average derived by dividing the area of the elevation of the building, as projected on a plane parallel to the direction considered, by the base width b.

(*b*) Towers having a plan area less than 25% of the plan area of the lower part of the building may be designed as separate structures for their own height, width, and weight. In this case the resulting total shear from the tower is to be applied at the top of the lower part of the building which shall be otherwise considered separately for its own height, width, and weight.

Sect. 09.—Design Requirements

(*a*) *Structural Frames.*—The lateral force-resisting capacity of any moment-resisting structural frame may be deducted from the total required lateral force to determine the required resistance of other lateral force-resisting elements, provided that such a frame has a resistance of at least 25% of the total lateral forces hereinbefore required, and provided that the building or structure as a whole shall be designed to resist a lateral force resulting from a coefficient C equal to not less than 0.02.

(*b*) *High Buildings.*—In buildings higher than 135 ft a moment-resisting frame shall be provided, designed to resist a lateral force resulting from a coefficient C equal to 0.01, or the wind force specified in Sect. 02, whichever is the greater.

(*c*) *Increase of Unit Stresses for Horizontal Forces.*—

1. For combined stresses due to lateral loads caused by earthquakes or wind, and for the vertical design dead and live loads as defined elsewhere, the permissible unit stresses may be increased one third, provided the section thus required is not less than that required for dead and live loads alone. For members carrying

stresses due only to lateral forces caused by earthquakes or wind the permissible unit stresses may be increased one third.

2. In foundations due consideration shall be given to the capacity of the soil under combined vertical and short time lateral loading, but any increase shall not exceed 100% of the normal bearing value.

(*d*) *Combined Axial and Bending Stresses in Columns.*—

1. Maximum allowable extreme fiber stress in columns at the intersection of a column with floor beams or girders, for combined axial and bending stresses, shall be the allowable bending stress for the material used. Within the center one half of the unsupported length of the column, the combined axial and bending stresses shall be such that

$$\frac{f_a}{f_{aa}} + \frac{f_b}{f_{ab}} \leqq 1$$

in which: f_a is the axial stress; f_{aa} is the allowable axial stress; f_b is the bending stress; and f_{ab} is the allowable bending stress.

2. When stresses are due to a combination of vertical and lateral loads, the allowable unit stresses may be increased as provided in Sect. 09(*c*).

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